Chapter 6
Verification of Design

1. Foundation Quality

Construction can cause defects in driven piles or drilled shafts. Unfortunately, an installed deep foundation is mostly below the ground surface and cannot be seen. The quality of the foundation should be verified to ensure adequate structural integrity, to carry the required load without a bearing capacity failure, to limit displacements of the structure to within acceptable levels, and to avoid unnecessary overdesign of the foundation. This chapter describes methods commonly used to verify the capability of the foundation to support a structure. These methods are nondestructive and usually permit the tested piles or drilled shafts to be used as part of the foundation.

a. Indicators of problem with driven piles. Piles driven into soils with variable stratification that show driving records containing erratic data, which cannot be explained by the construction method, indicate possible pile damage. Failure to reach the prescribed tip elevation or penetration rate also indicates pile damage. Other indicators include drifting of the pile off location, erratic driving unexplained by the soil stratification, and a sudden decrease in driving resistance or interference with nearby piles as indicated by sound or vibration. A pile can also be damaged during extraction.

b. Indicators of problems with drilled shafts. Most problems with drilled shafts are related to construction deficiencies rather than design. Such problems result from inadequate information of the subsurface soil and groundwater conditions provided to the contractor, inadequate clean-out including the presence of water in the excavation prior to concrete placement, inadequate reinforcement, and other complications during concrete placement. Drilled shaft failures may result from neglecting vertical dimensional changes in shrinking and swelling soil as those described in TM 5-818-7.

2. Driven Piles

Piles can be bent or sheared during installation and can cause a reduction in pile capacity. Piles can also undergo excessive tensile stresses during driving, specifically when soil layers have variable density or strength or when there is no significant end bearing resistance. Field test procedures such as standard penetration tests, pile driving analysis (PDA) with the wave equation, restrikes, and pile load tests can determine the ability of the pile to carry design loads. Refer to paragraph 4, Chapter 6, for guidance on load tests. Typically 2 to 5 percent of the production piles should be driven as indicator piles, at the start of construction at locations specified by the design engineer or at suspicious locations to confirm the capability of the driven piles to support the structure. PDA should also be performed during the driving of indicator piles and some static load tests performed to calibrate wave equation analyses. Table 6-1 illustrates an example procedure for verifying pile design. Analyses by wave equation and pile driving are presented.

a. Wave equation analysis. The penetration resistance in blows/feet (or blows/inch) measured when the pile tip has been driven to the required depth can be used to calculate the ultimate bearing capacity and verify design. Wave equation analyses can relate penetration resistance to the static ultimate bearing capacity.

(1) Computer program GRLWEAP. A wave equation analysis is recommended, except for the simplest projects when adequate experience and data already exist, for estimating the behavior of pile driving and confirming pile performance. This analysis may be accomplished using program GRLWEAP (Goble et al. 1988), Wave Equation Analysis of Pile Driving, licensed to WES. Program GRLWEAP and user's manual with applications are available to offices of the Corps of Engineers. GRLWEAP models the pile driving and soil system by a series of elements supported by linear elastic springs and dashpots with assumed parameters, Figure 6-1. Each dashpot and spring represent a pile or soil element. Information required to use this program includes indentification of the hammer (or ram) and hammer cushion used, description of the pile, and soil input parameters. Hammer selection is simplified by using the hammer data file that contains all the required information for numerous types of hammers. A simple guide for selection of soil input parameters to model the soil resistance force is provided as follows:

(a) The soil resistance force consists of two components, one depends on pile displacement, and theother depends on pile velocity. Pile displacement dependent resistance models static soil behavior, and it is assumed to increase linearly up to a limiting deformation, which is the quake. Deformation beyond the quake requires no additional force. The pile velocity component models depend on soil damping charactertistics where the relationship between soil resistance and velocity is linear and the slope of such relationship is the damping constant. Quake and damping constants are required for both skin friction and end-bearing components. Table 6-2 gives recommended soil parameters, which should be altered depending on local experience. The distribution of soil resistance between skin friction and end bearing, which depend on the pile and soil bearing strata, is also required. End-bearing piles may have all of the soil resistance in end

Table 6-1
Procedure for Verifying Design and Structural Integrity of Driven Piles

Step	Procedure
1	Complete an initial wave equation analysis selecting soil damping constants J_c, quakes ρ_u, distribution of soil resistance between skin friction and end bearing and the ultimate bearing capacity Q_u. Use the proposed pile and driving system. Adjust driving criteria as needed to reduce pile stresses and to optimize pile driving.
2	Drive indicator piles, typically 2 to 5 percent of the production piles, at locations specified by the design engineer using driving criteria determined by the wave equation analysis. Complete additional wave equation analysis using actual hammer performance and adjust for changes in soil strength such as from freeze or relaxation. Drive to various depths and determine penetration resistances with the PDA using the Case method to determine the static ultimate bearing capacity Q_u.
3	Restrike the piles after a minimum waiting period, usually 1 day, using the PDA and Case methods to determine actual bearing capacity that includes soil freeze and relaxation.
4	Perform CAPWAPC analysis to calibrate the wave equation analysis and to verify field test results. Determine Q_u, hammer efficiency, pile driving stresses and structural integrity, and an estimate of the load-displacement behavior.
5	Perform static load tests to confirm the dynamic test results, particularly on large projects where savings can be made in foundation costs by use of lower factors of safety. Dynamic tests may also be inconclusive if the soil resistance cannot be fully mobilized by restriking or by large strain blows such as in high capacity soil, intact shale, or rock. Static load tests can be significantly reduced for sites where dynamic test results are reliable.
6	Additional piles should be dynamically tested during driving or restruck throughout pile installation as required by changes in soil conditions, load requirements, piles, or changes in pile driving.
7	Each site is unique and often has unforeseen problems. Changes may be required in the testing program, type and length of pile, and driving equipment. Waivers to driving indicator piles and load testing requirements or approval for deviations from these procedures must be obtained from HQUSACE/CEMP-ET.

bearing, while friction piles may have all of the soil resistance in skin friction.

(b) A bearing-capacity graph is commonly determined to relate the ultimate bearing capacity with the penetration resistance in blow/feet (or blows/inch). The penetration resistance measured at the pile tip is compared with the bearing-capacity graph to determine how close it is to the ultimate bearing capacity. The contractor can then determine when the pile has been driven sufficiently to develop the required capacity.

(c) Wave equation analysis also determines the stresses that develop in the pile. These stresses may be plotted versus the penetration resistance or the ultimate pile capacity to assist the contractor to optimize pile driving. The driving force can be adjusted by the contractor to maintain pile tensile and compressive stresses within allowable limits.

(d) GRLWEAP is a user friendly program and can provide results within a short time if the engineer is familiar with details of the pile driving operation. The analysis should be performed by Government personnel using clearly defined data provided by the contractor.

(2) Analysis prior to pile installation. A wave equation analysis should be performed prior to pile driving as a guide to select properly sized driving equipment and piles to ensure that the piles can be driven to final grade without exceeding the allowable pile driving stresses.

(3) Analysis during pile installation. Soil, pile, and driving equipment parameters used for design should be checked to closely correspond with actual values observed in the field during installation. Sound judgment and experience are required to estimate the proper input parameters for wave equation analysis.

(a) Hammer efficiencies provided by the manufacturer may overestimate energy actually absorbed by the pile in the field and

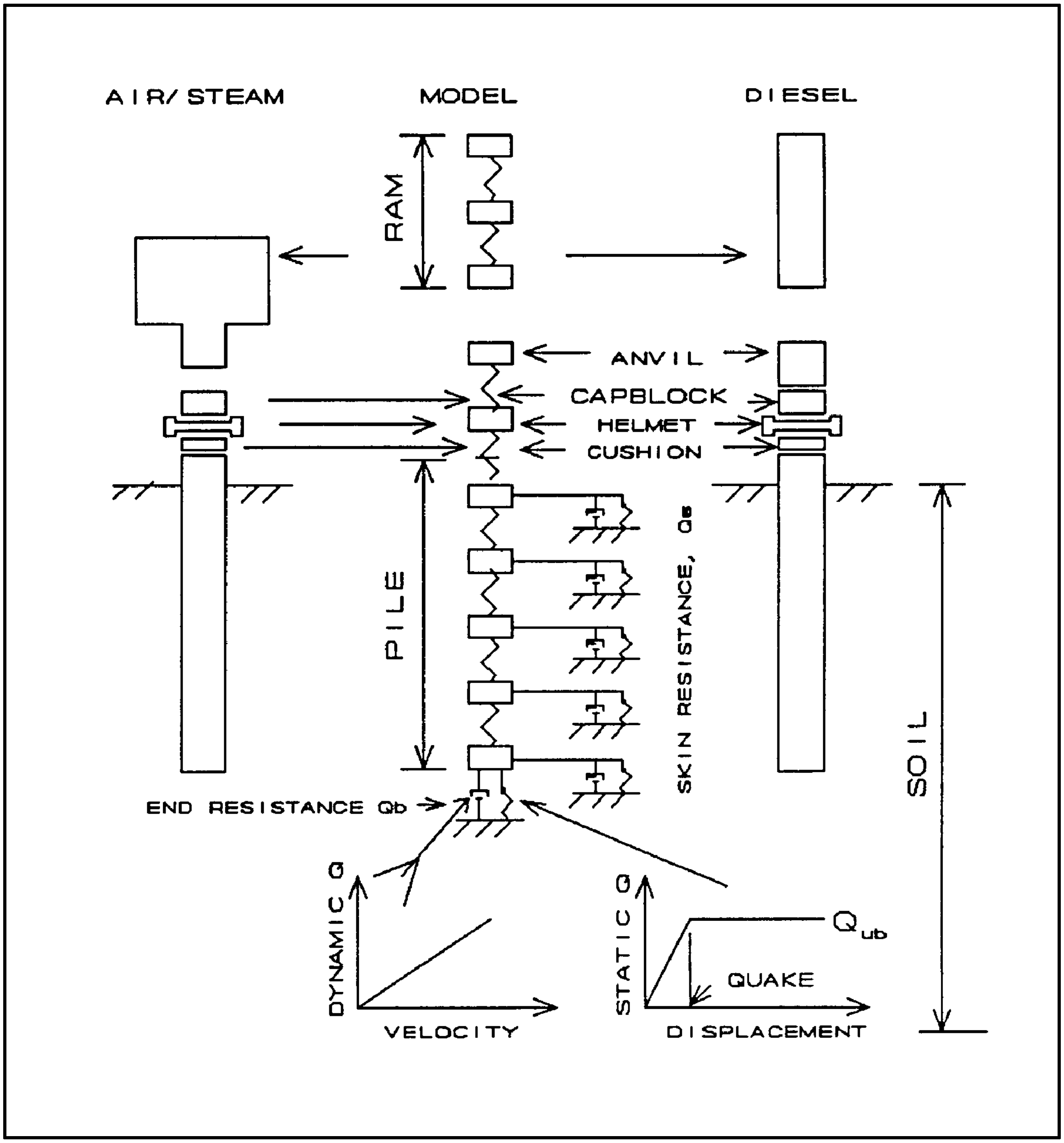


Figure 6-1. Schematic of wave equation model

may lead to an overestimate of the bearing capacity. Significant error in estimating ahmmer efficiency is also possible for driving batter piles. A bracket analysis is recommended for diesel hammers with variable strokes. Results of the PDA and ststic with variable strokes. Results of the PDA and static load tests described below and proper inspection can be used to make sure that design parameters are realistic and that the driven piles will have adequate capacity.

(b) Results of wave equation analysis may not be applicable if soil freeze (setup) occurs. Saturated sensitive clays and loose sands may lose strength during driving which can cause remolding and increasing pore water pressure. Densification of sands during driving contribute to a buildup of pore pressure. Strength regain is increased with time, after the soil freeze or setup. Coral sands may have exceptionally low penetration resistance during driving, but a reduction in pore pressure after driving and cementation that increases with time over a period of several weeks to months can contribute substantially to pile capacity. Significant cementation may not occur in several weeks.

(c) Penetration resistance is dense, final submerged sand, inorganic silts or stiff, fissured, friable shale, or clay stone can dramatically increase during driving, apparently from dilation and reduced pore water pressure. A "relaxation" (decrease) in penetration resistance occurs with time after driving. Driving equipment and piles shall be selected with sufficient capacity to overcome driving resistance or driving periodically delayed to allow pore water pressures to increse.

Table 6-2
Recommended Soil Parameters for Wave Equation (Copyright permission, Goble, Rausche, Likins and Associates, Inc. 1988)

	Damping Constants J_c, seconds/ft (seconds/m)		Quake ρ_u, inches (mm)	
Soil	**Skin**	**Tip**	**Skin**	**Tip**[1]
Cohesionless	0.05 (0.16)	0.15 (0.50)	0.10 (2.54)	B_b / 120
Cohesive	0.30 (0.90)	0.15 (0.50)	0.10 (2.54)	B_b / 120

[1] Selected tip quake should not be less than 0.05 inch. B_b is the effective tip (base) diameter; pipe piles should be plugged.

(d) The pile shall be driven to a driving resistance that exceeds the ultimate pile capacity determined from results of wave equation analysis or penetration resistance when relaxation is not considered. Driving stresses in the pile shall not exceed allowable stress limits. Piles driven into soils with freeze or relaxation effects should be restruck at a later time such as one or more days after driving to measure a more realistic penetration resistance for design verification.

(e) Analysis of the bearing capacity and performance of the pile by wave equation analysis can be contested by the contractor and resolved at the contractor's expense through resubmittals performed and sealed by a registered engineer. The resubmittal should include field verification using driving and load tests, and any other methods approved by the Government design engineer.

b. Pile driving analysis. Improvements in electronic instruments permit the measurement of data for evaluating hammer and driving system performance, pile driving stresses, structural integrity, and ultimate pile capacity. The required data may be measured and pile performance evaluated in fractions of a second after each hammer blow using pile driving analyzer equipments. PDA is also useful when restriking piles after some time following pile installation to determine the effects of freeze or relaxation on pile performance. The Case method (Pile Buck, Inc. 1988) developed at Case Institute of Technology (now Case Western Reserve University) is the most widely used technique. The CAPWAPC analytical method is also applied with results of the PDA to calibrate the wave equation analysis and to lead to reliable estimates of the ultimate static pile capacity provided soil freeze, relaxation, or long-term changes in soil characteristics are considered. The CAPWAPC method quakes and damping factors, and therefore, confirms input data required for the wave equation analysis.

(1) PDA equipment. PDA can be performed routinely in the field following a schematic arrangement shown in Figure 6-2. The system includes two strain transducers and two accelerometers bolted to the pile near its top, which feed data to the pile driving analyzer equipment. The oscilloscope monitors signals from the transducers and accelerometers to indicate data quality and to check for pile damage. The tape recorder stores the data, while an optional plotter can plot data. Digital computations of the data are controlled with a Motorola 68000 microprocessor with output fed to a printer built into the pile driving analyzer. The printer also documents input and output selections.

(a) The strain transducers consist of four resistance foil gauges attached in a full bridge.

(b) The piezoelectric accelerometers measure pile motion and consist of a quartz crystal that produces a voltage proportional to the pressure caused by the accelerating pile mass.

(c) Data can be sent from the pile driving analyzer to other equipment such as a plotter, oscilloscope, strip chart recorder, modem for transmitting data to a distant office or analysis center, and a computer. The computer can be used to analyze pile performance by the Case and CAPWAPC methods.

(2) Case Method. This method uses the force $F(t)$ and acceleration $\ddot{a}(t)$ measured at the pile top as a function of time during a hammer blow. The velocity $v(t)$ is obtained by integrating the acceleration. The PDA and its transducers were developed to obtain these data for the Case method.

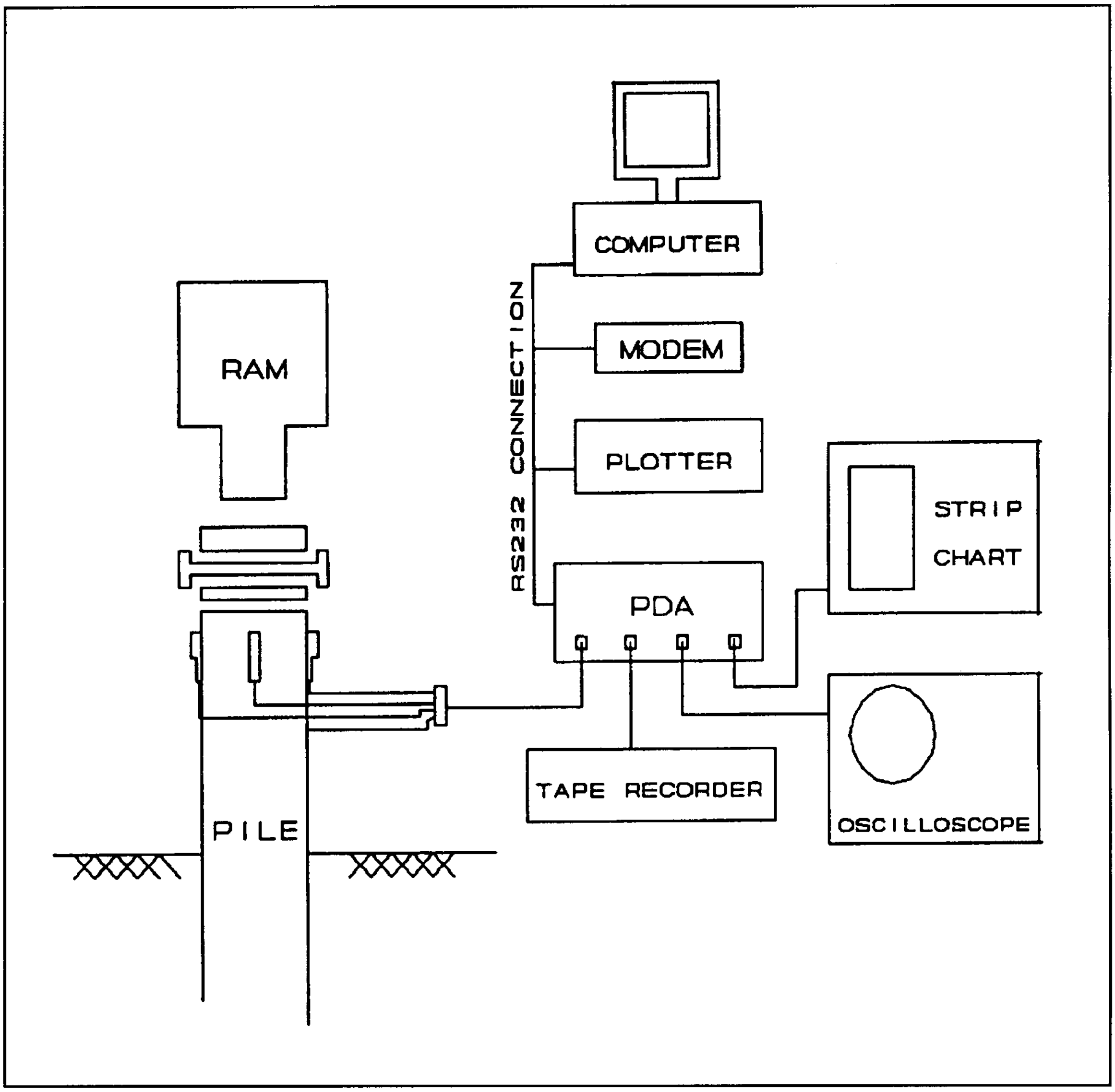


Figure 6-2. Schematic of field pile driving analyzer equipment

integrating the acceleration. The PDA and its transducers were developed to obtain these data for the Case method.

(a) The total soil resistance during pile driving R is initially calculated using wave propagation theory and assuming a uniform elastic pile and an ideal plastic soil as

$$R = \frac{F(t_1) + F(t_2) + Z_p[v(t_1) - v(t_2)]}{2} \qquad (6\text{-}1)$$

where

$F(t)$ = force measured by a strain transducer at a selected time t

t_1 = first selected time

$t_2 = t_1 + 2L/c$

Z_p = pile impedance, $M_p c/L$

$v(t)$ = velocity determined by integration of the accelerometers measured as a function of time

M_p = pile mass

c = wave transmission speed in the pile

t_1 is often selected as the time at the first maximum velocity. R is the sum of the static soil (displacement dependent), Q_u and the dynamic (velocity dependent) D components are of the capacity.

(b) Static soil capacity Q_u can be calculated from R by

$$Q_u = R - J_c(2Z_pV_{top} - R) \qquad (6\text{-}2)$$

where V_{top} is the velocity of the wave measured at the pile top at time t_1. Approximate damping constants J_c have already been determined for soils as given in Table 6-2 by comparing Case method calculations of static capacity with results of load tests. J_c can be fine tuned to actual soil conditions if load test results are available.

(c) Proper calculation of Q_u requires that the displacement obtained by integration of the velocity at time t_1, $v(t_1)$, exceeds the quake (soil compression) required for full mobilization of soil r esistance. Selection of time t_1 corresponding to the first maximum velocity is usually sufficient.

(d) A correction for ealy skin friction unloading causing a negative velocity may be required for long piles with high skin friction. The upper shaft friction may unload if the pile top is moving upward before the full resistance is mobilized. A proper correction can be made by adding the skin friction resistance that was unloaded to the mobilized soil resistance.

(e) Proper calculation to static resistance requires that freeze or relaxation effects are not present. Piles may be restruck after a waiting period such as 1 day or more to allow dissipation of pore water pressures.

(f) The driving force must be sufficient to cause the soil to fail; otherwise, ultimate capacity is only partially mobilized and the full soil resistance will not be measured.

(3) CAPWAPC method. This is an analytical method that combines field measured data with wave equation analysis to calculate the static ultimate bearing capacity and distribution of the soil resistance. Distribution of soil resistance, Q_u , and the pile load-displacement behavior calculated by the CAPWAPC method may be used to evaluate the damping constant J_c , quakes and soil resistances required in the Case method, and to confirm the determination of Q_u calculated using the Case method. The CAPWAPC method is often used as a supplement to load tests and may replace some load tests.

(a) The CAPWAPC method is begun using a complete set of assumed input parameters to perform a wave equation analysis. The hammer model, which is used to calculate the pile velocity at the top, is replaced by a velocity that is imposed at the top pile element. The imposed velocity is made equal to the velocity determined by integration of the acceleration. The CAPWAPC method calculates the force required to give the imposed velocity. This calculated force is compared with the force measured at the pile top. The soil input parameters are subsequently adjusted until the calculated and measured forces and calculated and measured velocities agree as closely as practical such as illustrated in Figure 6-3. The CAPWAPC method may also be started by using a force imposed at the pile top rather than an imposed velocity. The velocity is calculated and then compared with the velocity measured at the pile top. The CAPWAPC method is applicable for simulating static and dynamic tests.

(b) A simulated static load test may be performed using the pile and soil models determined from results of a CAPWAPC analysis. The pile is incrementally loaded, and the force and displacements at the top of the pile are computed to determine the load-displacement behavior. Actual static load test results can be simulated within 10 to 15 percent of computed results if the available static resistance is fully mobilized and time dependent soil strength changes such as soil freeze or relaxation are negligible.

(c) Dynamic tests with PDA and the CAPWAPC method provide detailed information that can be used with load factor design and statistical procedures to reduce factors of safety and reduce foundation cost. The detailed information on hammer performance, driving system, and the pile material can be provided to the contractor to optimize selection of driving equipment and cushions, to optimize pile driving, to reduce pile stresses, to reduce construction cost, and to improve construction quality. The foundation will be of higher quality, and structural integrity is more thoroughly confirmed with the PDA method because more piles can be tested by restriking the pile than can be tested by applying actual static loads. PDA can also be used to simulate pile load test to failure, but the pile can still be used as part of the foundation, while actual piles loaded to failure may not be suitable foundation elements.

3. Drilled Shafts

Drilled shafts should be constructed adequately and certified by the inspector. Large shafts supporting major structures are sometimes tested to ensure compliance with plans and specifications. Sonic techniques may be used to ascertain homogeneity of the foundation. Sonic wave propagation with receiver embedded in the concrete is the most reliable method for detecting voids or other defects. Striking a drilled shaft as ina large strain test with PDA and wave equation analysis is recommended for analysis of the ultimate pile capacity and load-displacement behavior as decribed above for driven piles. A large strain test may be conducted by dropping a heavy load onto the head of the shaft using a crane. Static load tests are commonly performed on selected shafts or test shafts of large construction projects to verify shaft performance and efficiency of the design.

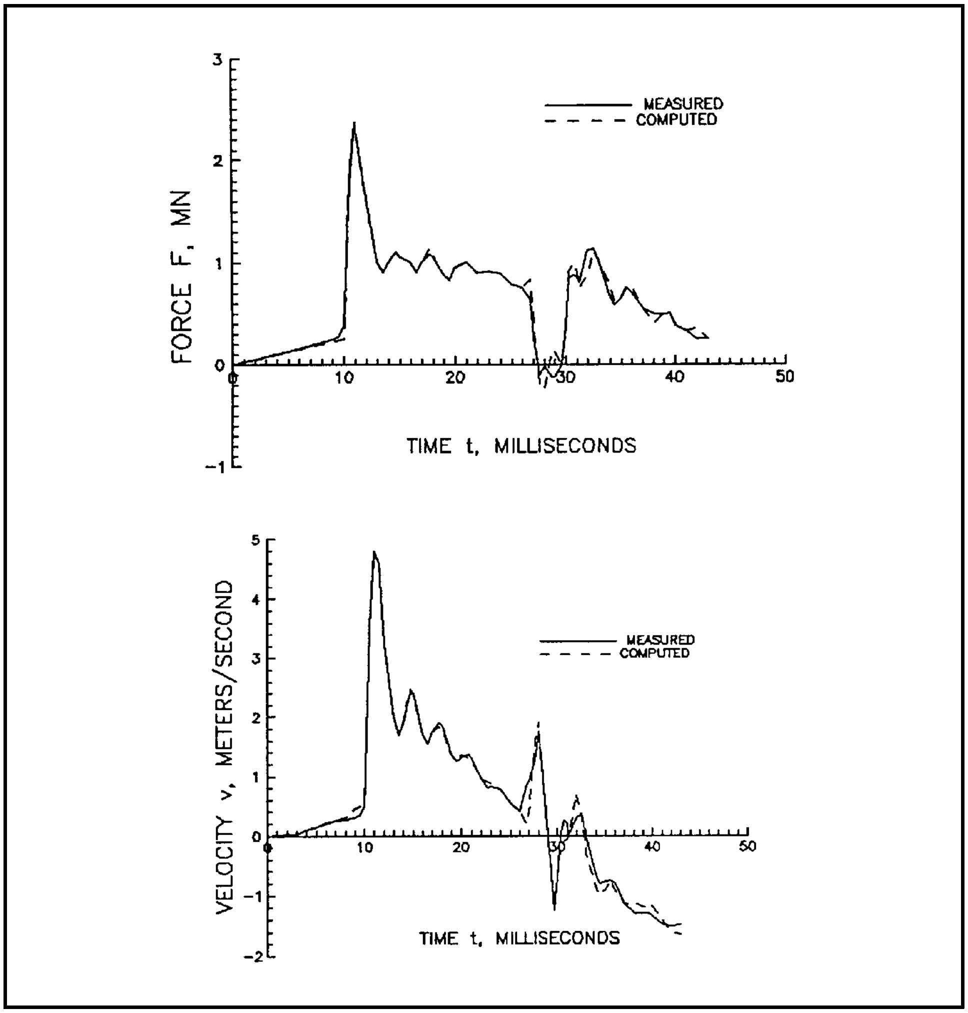


Figure 6-3. Example results of CAPWAPC analysis

a. Performance control. Continuous monitoring is essential to ensure that the boreholes are properly prepared to minimize loss of soil friction and end-bearing capacity and that the concrete mix is placed to achieve a continuous adequate shaft. Complete details of a drilled shaft construction control and an example of quality control forms may be found in FHWA-HI-88-042, "Drilled Shafts: Construction Procedures and Design Methods" and ADSC (1989) report, "Drilled Shaft Inspector's Manual." Construction and quality control include the following:

(1) Borehole excavation. Soil classification provided by all available boring logs in the construction area should be correlated with the visual description of soil or rock removed from the excavation. Any observed groundwater levels should also be recorded. Characteristics to be observed and determined include determined include location of the various strata, location and nature of the bearing stratum, and any seepage. The observer should also determine if the soil profile is substantially different from the one assumed for the design based on knowledge of the plans, specifications, and previous geotechnical analysis. The design engineer should be at the construction site during boring of the first holes to verify assumptions regarding the subsurface soil profile and periodically thereafter to check on requirements for any design modifications.

(a) Excavation details such as changes in the advance rate of the boring tool and changes in the soil cutting, groundwater observations, and bottom heave should be recorded. These details can be used to modify excavation procedure and improve efficiency in the event of problems as well as to provide a complete record for later reference.

Other important data include type of excavation (e.g., dry, cased, or slurry), time of initiation and completion of the boring, estimates of location of changes in the soil strata, and description of each soil stratum. Determine any evidence of pervious lenses and groundwater, problems encountered during excavating (e.g., caving, squeezing, seepage, cobbles, or boulders), and the location of the bearing stratum. A small diameter test boring from the excavation bottom can be made and an undisturbed sample recovered to test the bearing soil.

(b) The excavation should be checked for proper length, diameter, and underream dimensions. Any lateral deviations from the plan location and unintentional inclination or batter should be noted on the report and checked to be within the required tolerance. Provided that all safety precautions have been satisified, the underream diameter can be checked by placing the underream tool at the bottom of the excavation and comparing the travel of the kelly when the underreamer is extended to the travel when it is retracted in the barrel of the underream tool. Electronic calipers may be used if the excavation was made with slurry or the hole cannot be entered for visual inspection. Extreme safety precautions must be taken if an inspector enters an excavation to ensure no fall-in of material, and he should be provided with adequate air supply, communications and lifeline, and hoisting equipment. In the event of entry, a liner or casing should be in place to protect against fall-in. Fresh air may be pumped through hoses extending to the bottom. Minimum diameter of casing for personal inspection is 2 feet. An alternative to downhole inspection is to utilize ADSC drilled shaft inspectors manuals.

(c) Slurry used during excavation should be tested for compliance with mix specifications after the slurry is mixed and prior to placing in the excavation. These tests are described in Table 6-3 and should be performed by the Government and reported to construction management and the designer.

(d) The bottom of the excavation should be checked before placement of the reinforcement cage and concrete to ensure that all loose soil is removed, water has not collected on the bottom of open boreholes, and the soil is in the correct bearing stratum. Depth of water in an open borehole should be less than 2 inches. Casing should be clean, smooth, and undeformed.

(2) Placement of reinforcement. The reinforcement cage should be assembled prior to placement in the excavation with the specified grade, size, and number of bars. The cage should be supported with the specified horizontal stirrups or spirals either tied or welded in place as required to hold bars in place and prevent misalignment during concrete placement and removal of casing. The minimum spacing between bars should be checked to ensure compliance with specifications for adequate flow of concrete through the cage. The cage should be checked for placement in the specified position and adequately restrained from lateral movement during concrete placement.

(3) Concrete placement. The properties of the concrete mix and placement method must be closely monitored to avoid defects in the shaft. A record of the type of cement, mix proportions, admixtures, quantities, and time loaded on the truck should be provided on the delivery ticket issued by the concrete supplier. The lapse of time since excavation of the borehole and method of concrete placement, including details of the tremie used to place the concrete, should be recorded. Concrete slump should be greater than 6 inches and the amount of concrete placed in the excavation for each truck should be recorded. A plot of the expected quantity calculated from the excavation dimensions and the actual quantity should be prepared to indicate the amount and location of the concrete overrun or underrun. Excessive overruns or any underruns observed during concrete placement will require an investigation of the cause. Any unusual occurrence that affects shaft integrity should be described.

Table 6-3
Specifications for Bentonite Slurry

Property	Supplied During Excavation			Test Method
Density	Not less than necessary to bore shaft and less than 70 lb/cu ft	Mud density	–	Constant volume sample cup with lid connected to a balance gravity arm is filled with slurry so when placing the lid some slurry is forced out of a hole in the lid. Tap the edge of the cup to break up any entrained air or gas. Wipe excess slurry from the cup and lid. Place the balance arm into the fulcrum and move the rider on the balance arm to balance the assembly. Read the specific gravity from the scale on the balance.
Viscosity	30 to 50 sec	Marsh funnel	–	Place a finger over the bottom chute of the funnel and fill the funnel with slurry through a screen at the top of the funnel until the slurry level reaches the bottom of the screen (1 quart capacity). The slurry is allowed to flow from the funnel through the chute and number of seconds required to drain the funnel is recorded. Time measured is the viscosity.
Shear strength	0.03 psf to 0.2 psf (1.4 to 10 N/m^2)	Shearometer	–	The initial strength is determined by filling a container about 3 inches in diameter to the bottom line on a scale with freely agitated slurry. The scale is vertically mounted in the container. A thin metal tube is lowered over the scale and released. The tube is allowed to settle for 1 minute and the shear strength recorded on the scale reading at the top of tube . The 10-minute gel strength is determined in a similar manner except that 10 minutes is allowed to pass before the tube is lowered over the scale.
pH	9.5 to 12	Indicator paper	-	A pH electric meter of pH paper may be used.
Sand	2 % maximum by volume	API method	-	A specified amount of slurry is mixed in a marked tube. The content mixture is vigorously shaken, and all of it is then poured through a No. 200 mesh screen so that sand particles are retained on the screen. The sand particles are washed into a marked tube by fitting the large end of a funnel down over the top of the screen holder, then inverting the screen and funnel assembly. The tip of the funnel is fitted into the clear measuring tube and water sprayed from a wash bottle on the screen. The percent volume of sand is read from the marked measuring tube after the sand has settled.

b. Nondestructive tests. Routine inspection with nondestructive tests (NDT) using wave propagation shall be performed to check the quality of the installed drilled shafts. Additional special tests as indicated in the following paragraphs are performed if defects are suspected in some drilled shafts. Routine tests performed as part of the inspection procedure are typically inexpensive and require little time. Special tests to determine defects, however, are often time consuming, expensive, and performed only for unusual situations.

(1) Routine inspection tests. The most common routine NDT is to externally vibrate the drilled shaft by applying a sudden load as from a hammer or heavy weight dropped from a specified height. Signals from the wave are recorded by transducers and accelerometers installed near the top of the shaft or embedded in the concrete at some location in the length of the shaft. Access tubes may also be installed in the shaft for down-hole instrumentation to investigate the concrete between access tubes. Refer to FHWA-HI-88-042

for further information.

(a) The PDA procedure as discussed for driven piles may also be used for drilled shafts, even though it cannot be considered a routine test for NDT. The force-time and velocity-time traces ofo the vibration recorded on the oscilloscope caused by a dynamic load can be interpreted by an experienced technician to determine discontinuities and their location in the concrete.

(b) The wave pattern of large displacements caused by dropping sufficiently large weights from some specified height can be analyzed by the PDA procedure and CAPWAPC method to determine the ultimate bearing capacity and load-displacement behavior.

(c) Vibration from a hammer blow measured with embedded velocity transducers (geophones) can confirm any possible irregularities in the signal and shaft defects. The transducers are inexpensive and any number can be readily installed and sealed in epoxy-coated aluminum cases on the reinforcing cage with no delay in construction. The embedded receivers provide a much reduced noise level that can eliminate much of the requirement for signal processing.

(d) Forced vibrations induced by an electrodynamic vibrator over a load cell can be monitored by four accelerometers installed near the shaft head (Preiss, Weber, and Caiserman 1978). The curve of v_o / F_o , where v_o is the maximum velocity at the head of the drilled shaft and F_o is the applied force, is plotted. An experienced operator can determine the quality of the concrete such as discontinuities and major faults if the length of the shaft is known. Information below an enlarged section cannot be obtained.

(2) Access tubes and down-hole instruments. Metal or plastic tubes can be cast longitudinally into a drilled shaft that has been preselected for special tests. These tubes usually extend full length, are plugged at the lower end to keep out concrete, and are fastened to the rebar cage. Various instruments can be lowered down the access tubes to generate and receive signals to investigate the quality of the concrete.

(a) A probe that delivers a sonic signal can be inserted down a tube and signal receivers inserted in other tubes. One tube can check the quality of concrete around the tube or multiple tubes can check the concrete between the tubes.

(b) An acoustic transmitter can be inserted in a fluid-filled tube installed in a drilled shaft and a receiver inserted to the same depth in an adjacent tube. This test can also be performed on a drilled shaft with only a single tube using a probe that contains the receiver separated by an acoustic isolator. A single tube can be used to check the quality of concrete around the tube.

(c) A gamma-ray source can be lowered down one tube and a detector lowered down to the same depth in another tube to check the density of concrete between the source and detector. A change in the signal as the instruments are lowered indicates a void or imperfection in the concrete.

(3) Drilling and coring. Drilled shafts that are suspected of having a defect may be drilled or cored to check the quality of the concrete. Drilling is to make a hole into the shaft without obtaining a sample. Coring is boring and removal of concrete sample. Drilling and coring can indicate the nature of the concrete, but the volume of concrete that is checked is relatively small and drilling or coring is time consuming, costly, and sometimes misleading. The direction of drilling is difficult to control, and the hole may run out the side of the shaft or might run into the reinforcement steel. Experienced personnel and proper equipment are also required to ensure that drilling is done correctly and on time.

(a) Drilling is much faster than coring, but less information is gained. The drilling rate can infer the quality of concrete and determine if any soil is in the shaft. A caliper can measure the diameter of the hole and determine any anomalies.

(b) Coring can determine the amount of concrete recovery and the concrete samples examined for inclusions of soil or slurry. Compression tests can be performed to determine the strength of the concrete samples. The cores can also be checked to determine the concrete to soil contact at the bottom of the shaft.

(c) Holes bored in concrete can be checked with a television camera if such an instrument is available. A portion of a borehole can also be packed to perform a fluid pressure test to check for leaks that could be caused by defects.

(d) Defects of large size such as caused by the collapse of the excavation prior to concrete placement or if concrete is absent in some portion of the shaft can be detected by drilling or coring. Defects can be missed such as when the sides of a rock socket are smeared with remolded and weak material. Coring can also detect defects that appear to be severe but are actually minor. For example, coring can indicate weak concrete or poor material, or poor contact with the end bearing soil or rock in the region of the core, but the remaining shaft could be sound and adequately supported by the soil.

c. Load tests. The only positive way to prove the integrity of a suspected drilled shaft is to perform a load test. Drilled shafts are often constructed in relatively large sizes and load tests are often not economically feasible. Replacing a suspected drilled shaft is often more economical than performing the load test.

(1) Application. Load tests as described in paragraph 4, Chapter 6, shall be performed for drilled shafts when economically feasible such as for large projects. Results of load tests can be used to reduce the *FS* from 3 to 2 and can increase the economy of the foundation when performed during design.

(2) Preload. An alternative to load tests is to construct the superstructure and to preload the structure to determine the integrity of the foundation. This test must be halted immediately if one or more drilled shafts show more settlement than is anticipated.

4. Load Tests

Field load tests determine the axial and lateral load capacity as a function of displacements for applied structural loads to prove that the tested pile or drilled shaft can support the design loads within tolerable settlements. Load tests are also used to verify capacity calculations and structural integrity using static equations and soil parameters. Soil parameters can be determined by laboratory and in situ tests, wave equation and pile driving analysis, and from previous experience. Load tests consist of applying static loads in increments and measuring the resulting pile movements. Some aspects of load tests that need to be considered are:

a. Categories of load tests. Types of load tests performed are proof tests, tests conducted to failure without internal instrumentation, and tests conducted to failure with instrumentation. Proof tests are not conducted to a bearing capacity failure of the pile or drilled shaft but usually to twice the design load. Tests conducted to failure without instrumentation determine the ultimate pile capacity Q_u, but do not indicate the separate components of capacity of end bearing Q_{bu} and skin resistance Q_{su}. Tests with internal instrumentation, such as strain gauges mounted on reinforcement bars of drilled shafts or mounted inside of pipe piles, will determine the distribution of load carried by skin friction as a function of depth and will also determine the end-bearing capacity when conducted to failure.

b. Limitations of proof tests. Many load tests performed today are "proof" tests, which are designed to prove that the pile can safely hold the design load or to determine the design load. Proof tests do not determine the ultimate capacity so that the pile is often designed to support a higher load than necessary and can cause foundation costs to be greater than necessary. Proof tests are not adequate when the soil strength may deteriorate with time such as from frequent cyclic loads in some soils. Coral sands, for example, can cause cementation that can degrade from cyclic loads.

c. Selecting and timing load tests. Load tests are always technically desirable, but not always economically feasible because they are expensive. These tests are most frequently performed to assist in the design of major structures with large numbers of piles where changes in length, size, and type of pile and installation method can provide significant cost savings. The costs of load tests should be compared with potential savings when using reduced safety factors permitted with the tests. Factors to be considered before considering load test are:

(1) Significance of structure. The type and significance of a structure could offset the added cost of load tests for a complex foundation when the consequences of failure would be catastrophic.

(2) Soil condition. Some subsurface investigations may indicate unusual or highly variable soils that are difficult to define.

(3) Availability of test site. Testing should not interfere with construction. Load tests should be conducted early after the site is prepared and made accessible. The contractor must wait for results before methods and equipment can be determined and materials can be ordered. Advantages of completing the testing program prior to construction include discovery of potential and resolution of problems, determination of the optimum installation procedure, determination of the appropriate type, length and size of the piles. Disadvantages include increased design time to allow for load tests and testing conditions and data extrated from a test site used in the design may not simulate actual construction conditions such as excavation, groundwater, and fill. Problems may also occur if different contractors and/or equipment are used during construction.

(4) Location. Test piles should be located near soil test borings and installed piezometers.

(5) Timing. Load tests of driven piles should be performed after 1 or more days have elapsed to allow

dissipation of pore water pressures and consideration of freeze or relaxation.

d. Axail load tests. Axial compressive load tests should be conducted and recorded according to ASTM D 1143. The quick load test described as an option in ASTM D 1143 is recommended for most applications, but this test may not provide enough time for some soils or clays to consolidate and may underestimate settlement for these soils. The standard load test takes much longer and up to several days to complete than the quick load test and will measure more of the consolidation settlement of compressible soils than the quick load test procedure. However, neither the standard test nor the quick test will measure all of the consolidation settlement. The cyclic load test will indicate the potential for deterioration in strength with time from repeated loads. Procedures for load tests are presented:

(1) Quick load test. The load is applied in increments of 10 to 15 percent of the proposed design load with a constant time interval between load increments of 2 minutes or as specified. Load is added until continuous jacking is required to maintain the test load (plunging failure) or the capacity of the loading apparatus is reached, whichever comes first.

(2) Standard load test. Load is applied in increments of 25 percent of the design load and held until the rate of settlement is not more than 0.01 inch/hour but not longer than 2 hours. Additional load increments are applied until twice the design load is reached. The load is then removed in decrements of 50, 100 and 200 percent of the design load for rebound measurements. This is a proof test if no further testing is performed. A preferred option of the standard load test is to reload the pile in increments of 50 percent of the design load until the maximum load is reached. Loads may then be added at 10 percent of the design load until plunging failure or the capacity of the equipment is reached. This option is recommended to evaluate the ultimate pile capacity.

(3) Repeated load test. The standard load test is initially performed up to 150 percent of the design load, allowing 20 minutes between load increments. Loads are removed in decrements equal to the load increments after 1 hour at the maximum applied load. Load is reapplied in increments of 50 percent of the design load allowing 20 minutes between increments until the previous maximum load is reached. Additional load is then applied and removed as described in ASTM D 1143. This test is useful to determine deterioration in pile capacity and displacements from cyclic loads.

(4) Tension test. Axial tension tests may be conducted according to ASTM D 3689 to provide information on piles that must function in tension or tension and compression. Residual stresses may significantly influence results. A minimum waiting period of 7 days is therefore required following installation before conducting this test, except for tests in cohesive soil where the waiting period should not be less than 14 days.

(5) Drilled shaft load test using Osterberg Cell. Load tests are necessary so that the design engineer knows how a given drilled shaft would respond to design loads. Two methods are used to load test drilled shaft: the Quick Load Test Method described in ASTM D 1143 standard, and the Osterberg Cell Method.

(a) Unlike the Quick Load ASTM test method which applies the load at the top of the drilled shaft, the Osterberg cell test method applies the load to the bottom of the shaft. The cell consists of inflatable cylindrical bellow with top and bottom plates slightly less than the diameter of the shaft. The cell is connected to double pipes, with the inner pipe attached to the bottom and the outer pipe connected to the top of the cell (Figure 6-4). These two pipes are separated by a hydraulic seal at the top with both pipes extended to the top of the shaft. The outer pipe is used as a conduit for applying fluid pressure to the previously calibrated cell. The inner pipe is used as a tell-tale to measure the downward movement of the bottom of the cell. It is also used to grout the space between the cell and the ground surface and create a uniform bearing surface. Fluid used to pressurize the cell is mixed with a small amount of water - miscible oil. The upward movement of the shaft is measured by dial gauge 1 placed at the top of the shaft (Figure 6-4). Downward movement is measured by dial gauge 2 attached to the top of the inner pipe above the point where it emerges from the outer pipe through the hydraulic seal.

(b) After drilling the shaft, the Osterberg cell is welded to the bottom of the reinforcing cage, lifted by crane, and inserted carefully into the hole. After proper installation and testing, the cell is grouted by pumping a carefully monitored volume of grout through the inner pipe to fill the space between the cell and the bottom of the hole. When the grout is set, concrete is pumped to fill the hole to the desired level and the casing is pulled. After concrete has reached the desired strength, the cell is pressurized internally to create an upward force on the shaft and an equal and opposite downward force in end bearing. As pressure increases, the inner pipe moves downward while the outer pipe moves upward. The upward movement is a function of the weight of the drilled shaft and the friction and/or adhesion mobilized between the surface concrete and the surrounding soil.

(c) The dial gauges are usually attached to a reference beam supported by two posts driven into the ground a sufficient distance apart (i.e., 10 feet or two shaft diameters, whichever is larger) (Figure 6-4) to eliminate the influence of shaft movement during the test. The difference in reading between dial gauge 1 and dial gauge 2 at any pressure level represents the elastic compression of the concrete. The load downward-deflection curve in end bearing and the load upward- movement curve in skin friction can be plotted from the test data to determine the ultimate load of the drilled shaft. Failure may occur in end bearing or skin friction. At that point the test is considered complete. Osterberg cells can be constructed as large as 4 feet in diameter to carry a load equivalent to 6,000 tons of surface load.

(6) Analysis of capacity. Table 6-4 illustrates four methods of estimating ultimate capacity of a pile tested to failure. Three methods should be used when possible, depending on local experience and preference, to determine a suitable range of probable ultimate capacity. The methods given in Table 6-4 give a range of Q_u from 320 to 467 kips for the same test data.

(7) Effects of layered soils. Layered soils may cause the test piles to have a different capacity than the service piles if the test piles have tips in a different stratum. Consolidation of a cohesive layer supporting the tip load may also cause the load to be supported by another layer. The support of a pile could change from friction to end bearing or the reverse depending on the strata.

e. *Lateral load test.* This test is used to verify the stiffness used in design. The cyclic reduction factor used in design can be verified if the test pile is loaded for approximately 100 cycles. Some aspects of the lateral load test are:

(1) Monotonic and cyclic lateral load tests should be conducted and recorded according to ASTM D 3966. Tests should be conducted as close to the proposed structure as possible and in similar soil.

(2) Lateral load tests may be conducted by jacking one pile against another, thus testing two adjacent piles. Loads should be carried to failure.

(3) Groundwater will influence the lateral load response of the pile and should be the same as that which will exist during the life of the structure.

(4) The sequence of applying loads is important if cyclic tests are conducted in combination with a monotonic lateral load test. This may be done by first selecting the load level of the cyclic test using either load or deflection guidelines. The load level for the cyclic test may be the design load. A deflection criterion may consist of loading the piles to a predetermined deflection and then using that load level for the cyclic load test. Using the cyclic load level, the test piles would be cyclically loaded from zero loading to the load level of the cyclic load test. This procedure should be repeated for the required number of cycles. Dial gauge readings of lateral deflection of the pile head should be made at a minimum at each zero load level and at each maximum cyclic load level. The test pile should be loaded laterally to failure after the last loading cycle. The last loading cycle to failure can be superimposed on the initial loading cycle to determine the lateral load-deflection curve of the pile to failure.

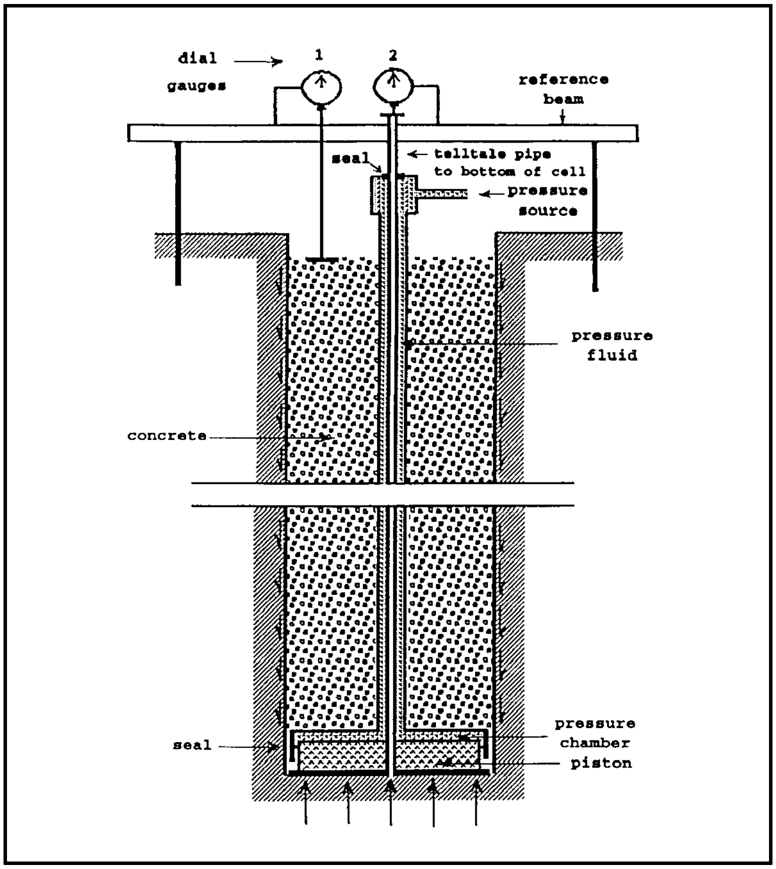
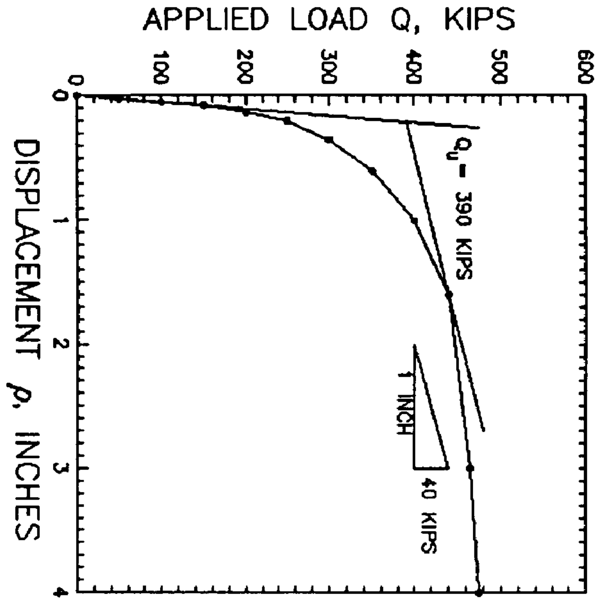


Figure 6-4. Typical Osterberg cell load test (from Osterberg 1995)

Table 6-4
Methods of Estimating Ultimate Pile Capacity from Load Test Data

<table>
<tr><th>Method</th><th>Procedure</th><th>Diagram</th></tr>
<tr>
<td>Tangent
(FHWA-RD-IR-77-8)</td>
<td>1. Draw a line tangent to the curve at the origin

2. Draw another line tangent to the curve at the point on the curve with slope equivalent to 1 inch for 40 kips of load

3. Ultimate bearing capacity is the load at the intersection of the two tangent lines</td>
<td>
</td>
</tr>
<tr>
<td>Limit Value
(Davisson 1972)</td>
<td>1. Draw a line with slope

$\frac{\pi B^2}{4L} \cdot E_p$ where

B = pile diameter, inches; E_p = Young's pile modulus, kips/inch²; L = pile length, inches

2. Draw a line parallel with the first line starting at a displacement of 0.15 + B/120 inch at zero load

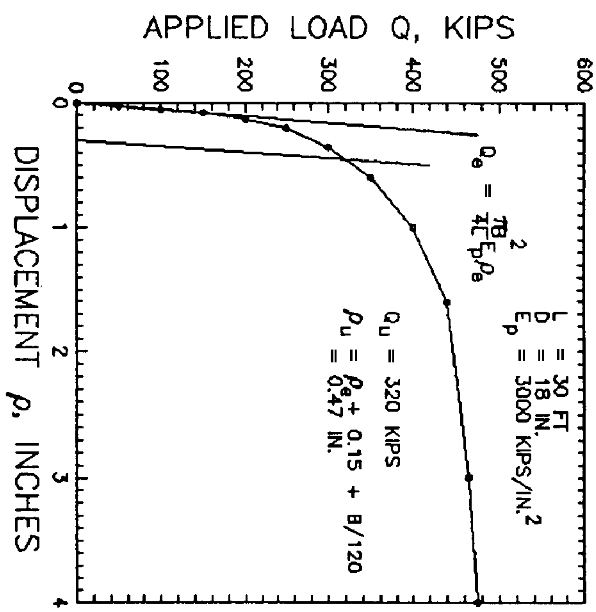
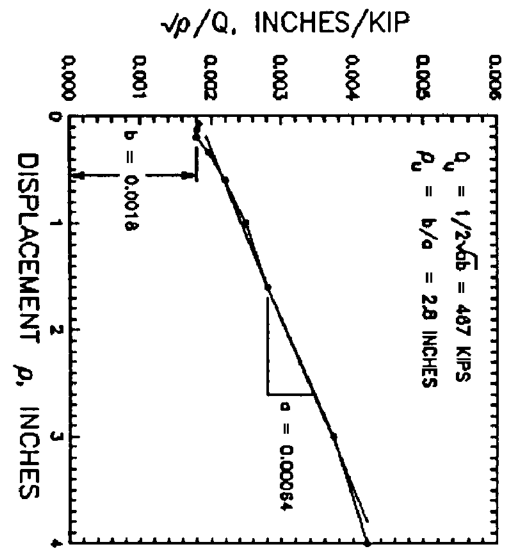
3. Ultimate bearing capacity is the load at the intersection of the load -displacement curve with the line of step 2</td>
<td>
</td>
</tr>
</table>

Table 6-4 (Concluded)

Method	Procedure	Diagram
80 Percent (Hansen 1963)	1. Plot load test results as $\sqrt{\rho}/Q$ vs. ρ 2. Draw straight line through data points 3. Determine the slope a and intercept b of this line 4. Ultimate bearing capacity is $Q_u = \frac{1}{2\sqrt{ab}}$ 5. Ultimate deflection is ρ_u = b/a	
90 Percent (Hansen 1963)	1. Calculate 0.9Q for each load Q 2. Find $\rho_{0.9Q}$, displacement for load of 0.9Q, for each Q from Q vs. ρ plot 3. Determine $2\rho_{0.9Q}$ for each Q and plot vs. Q on the chart with the load test data of Q vs. ρ 4. Ultimate bearing capacity is the load at the intersection of the two plots of data